SANTA CLARA VALLEY WATER DISTRICT

HYDROLOGY

DOCUMENTATION FOR THE NORTH CENTRAL FLOOD CONTROL ZONE

Prepared by

A. D. Saah, P.E.
Associate Civil Engineer
Project Development Branch

Under the Direction of

Managements of Water Supply Planning
and
Design and Construction Units

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June 1978

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INTRODUCTION

This report summarizes the work completed on the hydrology within the Santa Clara Valley Water District's North Central Flood Control Zone. Presented in the report are recommended design flows for Calabazas, Saratoga and San Tomas Creeks and each of their respective tributaries. They are divided into several hydrologic studies each containing a table of contents, a general information about the physical and hydro-meteorological features of the basin, and a discussion of the procedures followed. This last item includes the analysis of recorded data, as well as recorded and reconstituted data. It also includes a comparative analysis of the regional and site specific statistics.

The choice of the final design flows is based on findings from the above results. It is the intent of this report, as well as other similar reports, to present a documentation of the hydrologic assumptions made together with the steps taken in the mathematical calculations. Each creek was originally treated separately. Inasmuch as the study has been combined in terms of flood control zones, some redundancy is to be expected.

The following is a list of the tributaries analyzed in these studies:

Calabazas Creek Hydrology

Calabazas Creek
Prospect Creek
Regnart Creek
Rodeo Creek

Saratoga- San Tomas Creeks Hydrology

San Tomas Creek
Saratoga Creek
Smith Creek
Wildcat Creek

SUMMARY OF METHODS AND PROCEDURES

General

The District has established its flood protection criterion for watershed areas greater than four square miles based on flows which has a 1% chance of being equaled or exceeded in any one year. For watershed areas between one and four square miles the criterion is the 2% flood and for watershed areas between 360 acres and one square mile it is the 4% flood.

The accompanying reports represent the 1977-78 update of the design flood flows for use in the design of flood control facilities in Santa Clara County. They present documentation for the calculation of the 1% and 10% design flood flows. Other design values (2% and 4%) are not presented herein; they could be obtained upon request. These values were calculated using the District's methods and procedures as augmented and modified by the recommendations of the panel of expert hydrologists* convened to review the methods in 1976 (Ref. 8). It was assumed in the derivation of these design flows that the channel system is free from major constrictions or significant flood storage (reservoirs are handled separately) and that the floodwaters at the design level are completely contained within channel banks or relatively narrow floodway limits.

The District's method of determining the design peaks and volumes of floods utilized all available sources of information to derive the most accurate values possible. In general, this method began with the use of Regional Regression Equations to compute peaks and volumes. These peaks and volumes were compared with the results of an analysis of reliable recorded streamflow data when such exist. The values were also compared with the results of a rainfall-runoff model before the final "design flow" classification is given. Where design flows are presently affected by reservoirs or by urbanization, special analyses accounting for their effects were performed. Finally, the design flows were compared with the results of other studies and with previously determined design flows. The results of such comparison are useful to further define the design values.

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*The panel members were Leo Beard, Dave Dawdy, Neil Grigg, Douglas L. James and Vujica Yevjevich. Their report, "Review of Basic Hydrology Methodology for Flood Control" Santa Clara Valley Water District, 1976, is available from the District library.

Hydrographs were derived utilizing the Corps of Engineers' standard project storm and the Clark Synthetic Unit Hydrograph technique. The peaks and volumes of these hydrographs were then adjusted to agree with those values derived from regional or local statistics. The flood hydrographs obtained this way are considered representative for use in the design of most flood control facilities. However, where flood traps (small volume peak skimmers) are being considered, a sensitivity analysis of the design relative to hydrograph shape should be conducted.

Following is a more detailed disucssion of these methods. Full documentation of the analyis of each stream is available in separate reports.

Regional Regression Analysis

The backbone of the District's hydrology is the application of the Regional Regression Equations (1976) for the determination of flood peaks and volumes. These equations were obtained by analyzing recorded streamflow data (peak flows and their respective volumes) from stations within and outside the District. The data were correlated, expanded, and the resultant statistics adjusted to obtain the dependent variables (mean, standard deviation, and 1% flow). These statistics were then regressed to the physical characteristics of their respective basins (independent variables) by means of a mathematical process called Multiple Linear Regression. The regression equations chosen had minimal standard errors of estimates and maximum determination coefficients. These equations are applicable to all unurbanized locations within the District to generate statistics for the determination of design flood peaks and volumes.

Gaged and Ungaged Locations

At gaged locations the statistics from the regression equations were compared with the site-specific statistics obtained from recorded data when such data were usable. The statistics from recorded data were obtained using the methods presented in the Water Resources Council Bulletin #17 (Ref. 2). If there were significant differences between the regional and site-specific results, then new statistics (weighted with respect to their respective standard errors) were calculated. These statistics, together with a Pearson Type III distribution a -0.6 regional skew coefficient or a weighted one, were used to determine the frequency regime.

At ungaged and rural locations, the results of the regression equations are considered to provide the best available information and, hence, are to be used for analysis and design.

Techniques Used for Developing Flood Hydrographs

Pattern runoff hydrographs are obtained from rainfall using Clark's Synthetic Unit Hydrograph method (as developed by the Corps of Engineers' HEC-1 computer program). This rainfall runoff transformation involves the determination of loss rates, synthetic unit hydrograph parameters, spatial distribution of rainfall over the watershed and temporal distributions of rainfall rates within one storm. The first two items are usually obtained from projections of optimized results of recorded rainfall and its respective runoff data. The averaging of rainfall over the basin is usually done by the regular Theissen weighing operations. The temporal distribution is determined generally by using the Corps of Engineers' Standard Project Storm for Northern California.

If the peaks and volumes of the pattern hydrographs obtained are drastically different from the regional or site-specific results, then the reasons were investigated and, if justified, further adjustments were made. In this study, the rainfall-runoff-generated peaks and volumes are found to generally agree with regional or statistical values when average rainfall and area configurations were present. To avoid the uncertainties inherent in the rainfall-runoff transformation, the peaks and volumes generated from this transformation are balanced by regional values.

The balanced hydrographs obtained above are then combined with other hydrographs and routed downstream. The routing was done using linear methods (e.g., Muskingum by computing k when x = 0) if routing through channel system, or nonlinear methods (modified Pulse) if significant storage is available.

Rainfall distribution, time of concentration and the various routing operations are the items that define the shape of pattern hydrographs, with rainfall distribution being the foremost. The effect of the shape of these hydrographs, given constant peaks and volumes, may become significant when flow trapping is being considered as a flood protection measure. An in-depth analysis of the shape was not attempted as a part of this work.

Regulated Watersheds

In regulated watersheds, regulated by reservoirs, the design flows are assumed as inflows to the reservoirs. The initial storage levels of the reservoirs that coincide with the design flows of certain frequencies are computed from the stage-duration frequency (S.D.F.) analysis of daily recorded reservoir volumes. The Corps of Engineers' Coincidental Frequency Analysis (discussed in Leo Beard's "Statistical Methods") procedures were used to solve this compounded probability problem. At most reservoirs, the S.D.F. curves after 1968 were used because, since that time, the reservoirs have retained more water in storage longer to enhance recreation.

Urban Hydrology

The above procedures are applicable mainly to rural watersheds. For urban watersheds, the methodology changes and the regional regression equations do not apply.

Urban hydrology deals with the effects due to two major changes in the watershed, namely:

1. Increased imperviousness
2. Channelization

The increase in imperviousness is caused by rooftops, streets, parking lots, driveways, and other man-made ground surface cover. Channelization includes the flow of water in gutters, storm drains, improved channels, etc. An increase in the imperviousness tends to reduce the loss rates and increase the runoff volumes and peaks, while channelization may either increase or decrease the peaks. It depends on the magnitude of the flood and the storage-discharge capacity of the various channels or storm drain systems that carry the flood.

To determine the effect of increased imperviousness, the urban area of every watershed was divided into subareas tributary to the major storm drains. These subareas are then transformed into equivalent rectangular subareas where the length is equal to the length of an equivalent street. The ratio of imperviousness (which is a measure of land use) is then used to subdivide this equivalent subarea into two sub-subareas of pervious and impervious parts.

In order to calculate the separate inflow hydrographs from the two sub-subareas, the unit hydrograph procedure was employed using separate times of concentration for both the pervious and impervious parts. The routing coefficient is calculated from the length and width of these areas. The loss rates, which are the third set of parameters needed for the computation of the one percent flood hydrograph, are assumed to be 30 percent for the pervious part and five percent for the impervious part. Since most of the floodwater from urban areas is conveyed to flood control channels by the storm drain system, these hydrographs are routed through the storm drain system utilizing the modified Pulse routing method. This routing requires storage-discharge curves for the system of streets, gutters, manholes, and storm drains. Two actual urban areas, one of low-density residential and another of high-density residential and commercial, were used to develop equivalent generalized storage facilities and storage-discharge curves. The resultant curves were transformed to reflect unit areas, and then compared with each other. Since the two curves are very similar, an average smooth curve was chosen and used.

Because most of the urban area in Santa Clara Valley is in the flatland and similar storm drain design criteria were used throughout, it is assumed that the unit storage-discharge curve that was obtained from the above study is applicable to all the urban areas in the Valley.

The above method is detailed in a paper presented before the National Symposium on Urban Hydrology in 1976.

Upper 80 Percent Confidence Limits

As a District policy, the upper 80 percent confidence value for the 1% design flow rate is used as one input to define freeboard criteria. It is defined as the value at which there is a 90 percent chance that the design flow is equal to less than the given value. It relates to the standard errors associated with the statistical inferences of information about populations from very limited samples. These values are calculated using the Non-Central Student "t" Distribution as shown in the Water Resources Council Bulletin #17.

Previous Studies and Coordinated Results

As a part of the design flood flow determination procedure, all available hydrologic analyses done on any watershed were reviewed and considered.

These previous analyses were sometimes the result of an in-house effort and other times the result of efforts of other agencies.

Because of the nature of random events, especially in the case of small samples, the results obtained from previous analysis were not necessarily congruent with the new values. Therefore, an in-depth search of the adequacy of the various results and methods involved was done.

As a result of this study, some of the adopted values were supported not only by District-developed methods and techniques but also by those of the Corps of Engineers and others.

Further Information

The attached information is only a summary report of that available. Additional information, available upon request from the Hydrologic Library, is listed below:

1. Hydrographs, either plotted or numerical, at all locations.

2. Runoff from intermediate areas such as urban runoff.

3. Recorded streamflow data at most gaged locations.

4. Stochastically estimated data at most gaged stations.

5. A full spectrum of statistical analysis of the above data which include the investigation of outliers, the incorporation of historic flows, and the adjustment of statistics and probabilities.

6. Statistical analysis of rainfall data on all gaged locations.

7. Complete descriptions and documentation of computations done for other streams in the County.

8. Other supporting documents of the methods like the 1976 Panel of Experts Report, the 1976 Regional Regression Study, Paper on Urban Hydrology and Reports by Zone, discussing the hydrology in detail.

ACKNOWLEDGEMENTS

In an effort to accelerate the District's hydrology study schedule to match the requirements of the Federal Insurance Administration, George S. Nolte and Associates performed the hydrology study of Calabazas Creek under District direction.

E. S. Watson, Division Engineer, was very instrumental in directing this work and reviewing this report.

Randy Talley compiled the data needed for parts of this work and drafted parts of this report.

CALABAZAS CREEK HYDROLOGY

DETERMINATION OF DESIGN FLOWS

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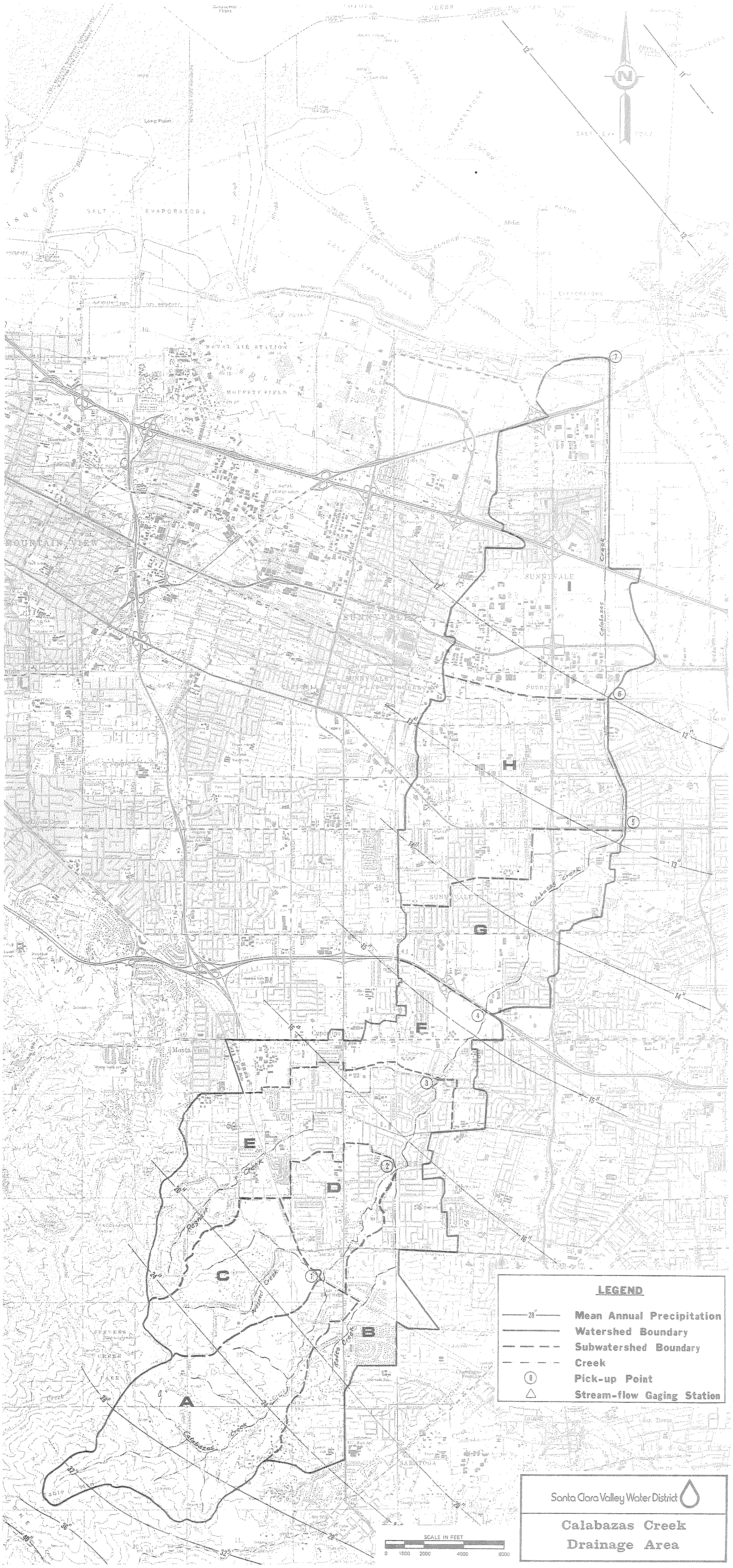
GENERAL INFORMATION

Calabazas Creek Watershed originates at 1,920 feet above mean sea level in the Santa Cruz Mountains and flows north through the Cities of Sunnyvale and Santa Clara (see Figure 1-1). It meets San Tomas Aquino Creek at sea level near San Francisco Bay. The total drainage area is 20.7 square miles, of which 2.8 square miles are rural. The basin receives an average annual rainfall ranging from 35 inches in the headwaters to 12 inches near the Bay. There are no reservoirs on Calabazas Creek which function as flood control facilities.

The objective of this report is to document the hydrology work done to determine the design flows. It will discuss the use of the required regression equations and the analysis of recorded data, together with the special treatment of urban areas.



Santa Clara Valley Water District

Calabazas Creek Drainage Area

Figure 1

DISCUSSION OF PROCEDURES

Basin Characteristics

The Calabazas Creek Watershed is divided into two major watersheds: Calabazas rural, which includes subareas A and C, and Calabazas urban, which includes subareas B, D, E, F, G, H and I (Figure 1-1). The subareas have the following basin characteristics:

Location	Sub-Area	*Drainage Area (Sq. mi.)	Mean Annual Precip. (in.)	Lls $(mi^2/ft/ft^{1/2})$	Tc (Hours)
Calabazas Creek U/S Prospect Crk	A	2.85	26.1	27.5	.79
Prospect Creek U/S Calabazas Crk	C	1.44	21.5	11.9	.54
Calabazas Creek U/S Rodeo Creek	D	AI - .58 AP - .38	18.0	-	.25 2.50
Rodeo Creek U/S Calabazas Creek	B	AI - .57 AP - .86	19.9	-	.25 3.30
Regnart Creek U/S Calabazas Creek	E	AI - 1.84 AP - 1.57	18.0	-	.25 2.70
Calabazas Creek at I-280	F	AI - 1.13 AP - .37	15.6	-	.25 2.10
Calabazas Creek at El Camino Real	G	AI - 1.46 AP - .65	14.0	-	.25 2.30
El Camino Storm Drain U/S Calabazas Creek	H	AI - 1.94 AP - 1.29	13.0	-	.25 2.50
Calabazas Creek at Slough	I	AI - 2.58 AP - 1.16	11.8	-	.25 2.30

*AI = Impervious Urban Area
AP = Pervious Urban Area
LLS = Basin Factor

In rural areas, the preliminary evaluation of design flows from regional regression equations is determined using the drainage areas, mean annual precipitation and basin factors. These preliminary values are compared with peak flows obtained from rainfall by employing the unit hydrograph theory and HEC-1 program. Although this latter technique is used mainly to define design hydrographs, the resultant peak flow is compared and balanced against the regional one. As will be shown later, the end result of the above procedure is further compared with results from the site specific analysis of recorded data if available.

Urban areas are treated based on the District's method of urban hydrology (Ref. 4). The density of land use is determined by the ratio of impervious to pervious parts.

Recorded Data and Application of Regional Regression Equations

Inherent in the District hydrology method are checking procedures that utilize additional data wherever availabe. Streamflow records, stochastically-generated data, or data obtained from rainfall, are utilized. Following is a discussion of some of these procedures and their application to the Calabazas Creek.

Streamflow Records

There is one stream gage (SCVWD #1 drainage area = 4.63 sq. mi.) in the 21 square mile drainage area which has operated at three alternate locations since its installation. The gage is maintained by the Santa Clara Valley Water District and is currently located near Rainbow Drive. Having records since 1946, it was used as one of the base stations in the Santa Clara Valley Water District Regional Regression Studies (Ref. 1). The highest annual peaks, as recorded by the Santa Clara Valley Water District Stream Gage on Calabazas Creek near Rainbow Drive*, are:

1.	1,800 cfs	December 22, 1955
2.	834 cfs	January 1, 1970
3.	720 cfs	January 24, 1967
4.	635 cfs	January 11, 1952
5.	635 cfs	January 30, 1968

This study analyzes the data up until 1976, a total of 30 years of record. In January, 1978 the recorded peak flow was 1,342 cfs which did not drastically affect the already used statistic.

---

*The stream gage was located at SPRR from 1945-1958; at Wardell Road from 1958-1966; and at Rainbow Drive from 1966 to the present.

Analysis of the 30 years of streamflow records produced the following statistics:

Mean (m) = 2.373

Standard Deviation (σ) = 0.449

Skew Coefficient (g) = -.53 +

1% Peak Flow (1%) = 1,750 cfs

The skew coefficient is difficult to determine reliably for stream gages which have a short period of record. The Water Resources Council (Ref. 2) recommends weighting the skew (g) if $25 \leq N \leq 100$, to produce an adopted skew as follows:

$$\text{Adopted weighted } g = \frac{N-25}{75} * \text{Recorded } g + \left(1 - \frac{N-25}{75}\right) * \text{regional } g$$

(where N is the number of recorded annual peak flows and g is the skew coefficient regional g = .6).

Based on the above equation and because the station has 30 annual peak flows, the station's weighted skew is approximately equal to -.6. The consistency of the record is uncertain due to the effects of urban encroachment in the upland region of the watershed and the changes of station location. It was assumed, however, that enough randomness is available in the record for a statistical analysis. The results of the regional equations alone and the station data alone, using the regional skew, are tabulated below:

		Regional Equations	Station Statistics
m	=	2.332	2.373
σ	=	.493	.449
g	=	-.6	-.6
1% (cfs)	=	1,810	1,650

These regional results, among others, are plotted along with the station data on Figure 1-2.

September 1976

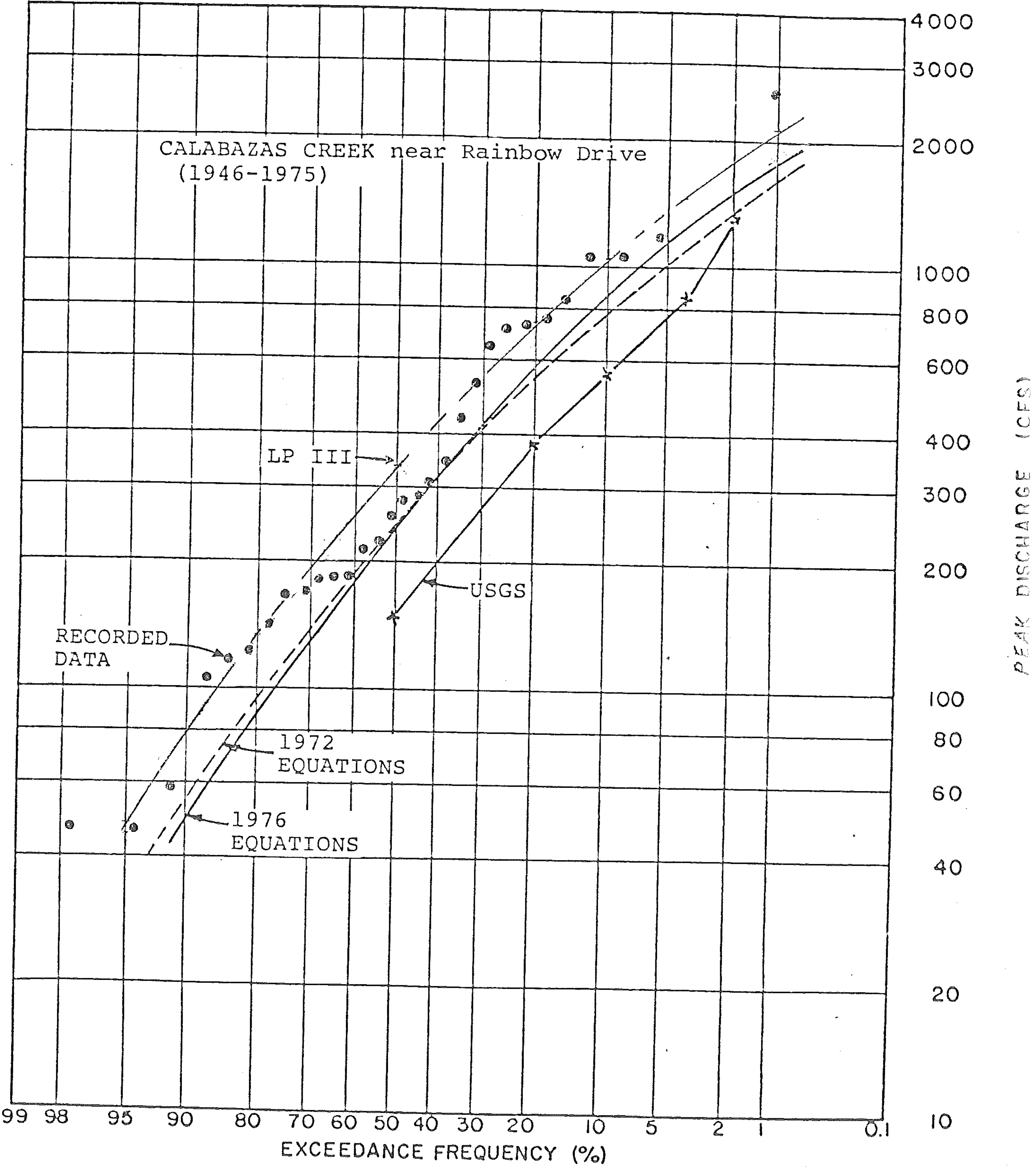


FIGURE 1-2

Comparisons with Similar Watersheds

Comparisons between frequency curves and maximum flood data of the watershed being investigated and those in a hydrologically similar region are useful for identification of unusual events and testing the reasonableness of the flow-frequency determinations.

Calabazas Creek gage was used as a base station in the regional regression analysis. From that study (Ref. 1), the 30 years of record (1946-1975) were correlated in a data matrix from 20 other streamflow stations with up to 60 years of record. The correlation enhanced the Calabazas gage statistics to be equivalent to 55 years of record. The following table provides adjusted statistics calculated from the 20-station/60-year recorded and synethetic data matrix.

	Adjusted Statistics
n (Years)	55
m	2.479
σ	.449
g	-.6
1% (cfs)	2,100

Therefore, three sets of statistics are available; regional, station's statistical and adjusted. The adjusted values are considered an improvement over the station data. Therefore, to include all information, the following weighted statistics (weighted with respect to the equivalent years of record as recommended in Refs. 2, 5, and 8) from regional and adjusted results were considered.

$$\text{Weighted } 1\% = \text{Antilog } \frac{[\log\ (1810)\ 15+\log\ (2100)\ 55]}{15 + 55} = 2{,}030 \text{ cfs}$$

The difference between the regional and weighted results is only 12%. Therefore, for consistency throughout the area, the use of the regional statistics without adjustment was considered appropriate.

Flood Estimates from Precipitation

Flood flows in streams are affected by the combinations of prior rainfall, soil condition, and available storage capacity. Under proper conditions, a storm of a certain intensity may cause a flood of larger size than what is expected. This is obvious as the dates of the maximum recorded one-day rainfalls, compiled in Table 1 -1 for several precipitation stations in the San Tomas drainage, do not necessarily fall on the same days as the larger floods. Maximum one-day rainfalls were used as they were readily available. The maximum 24-hour rainfall would be approximately 13 percent higher.

The mean annual precipitation (M.A.P.) reflects well the integrated effects of availability of atmospheric moisture, of elevation and topography, and of efficiency of the orographic mechanism, which are significant parameters of storm rainfall. It was, therefore, possible to develop a relationship of storm values vs. M.A.P. for different durations and frequency using data collected from rainfall stations in and around the County. The 24-hour rain with an exceedance frequency of 1% was used as the design storm.

The unit hydrograph theory was employed to convert rainfall to runoff (Ref. 3). Clark's synthetic unit hydrograph was used with the Corps of Engineers' 24-hour standard storm distribution and a constant loss rate. It has been found that a 30 percent loss rate applied to the 1% storm closely approximates the one percent flood on watersheds with average hydraulic characteristics.

The 1% peak flow rate at the Santa Clara Valley Water District gage is 1,800 cfs using this approach. This is an approximate figure and a parameter optimization technique would be necessary to refine it further; however, it does provide a guide to prevent gross errors.

Table 1-1

Records of Maximum Rainfall

Calabazas Creek

			Maximum Annual Recorded 1-Day Rainfall	
#5	Alviso (Leslie Salt)	1	3.54	January 12, 1952
	Elevation - 6.56'	2	2.40	December 24, 1955
	1941-1974	3	2.20	November 29, 1970
#20	Cox Avenue	1	4.70	December 22, 1955
	Elevation - 374.8'	2	4.26	January 21, 1967
	1955-1974	3	4.21	January 29, 1968
			1.61	March 5, 1978

OTHER STUDIES - PAST AND PRESENT

The hydrology of Calabazas Creek has been studied in the past. Values currently listed in the District's Manual of Design Flows dated 1971 (Ref. 6) are a conglomeration of application of several versions of regional regression equations.

The past regional equations, as developed in 1972, were used until revised in 1976. How well each set of equations represents the Calabazas Creek Watershed at the gaging station near Rainbow Drive is shown graphically in Figure 1-3, where each flow frequency curve is plotted in comparison with the recorded data at the Calabazas Creek stream gage. The U. S. Geological Survey has developed its own set of regression equations for the San Francisco Bay Region (Ref. 7). These equations were used to develop the USGS flow frequency curve in Figure 1-3.

SEPTEMBER 1976.

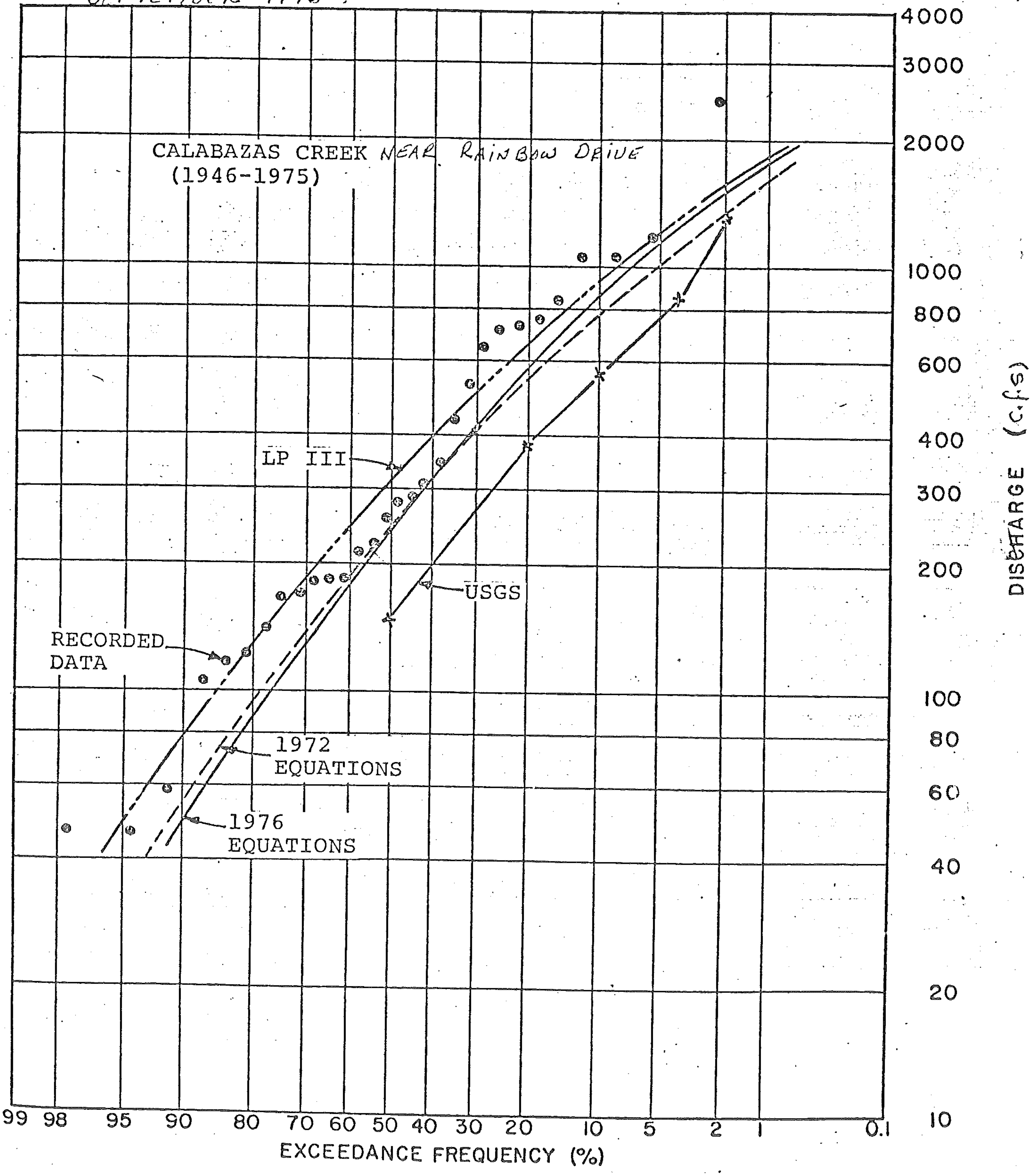


FIGURE 1-3

CONCLUSIONS

This report represents a summary of all the work performed in the study of the Calabazas Creek hydrology.

The flow rates have been computed from the 1976 regional regression equations, checked against statistical analysis of systematic streamflow records, compared with hydrologically similar watersheds, generated from rainfall, and contrasted with past studies of our own and of others.

The 1% design flow (1,800 cfs) obtained from the regional equations compares favorably with the flow values obtained from the Log Pearson Type III analysis of the station record (1,650 cfs) and the Army Corps of Engineers (1,800 cfs). Station statistics, combined with synthetic data, show a 1% flow of 2,100 cfs. Since weighting the regional statistics with the recorded and synthetic data matrix gives a 1% flow of 2,030 cfs, the results obtained from the regional equations are considered close enough to the weighted results (difference is 12%). Therefore, the regional results were chosen and used here for homogeneity with other non-instrumented locations.

The 1% and 10% design flows for Calabazas Creek are summarized in Tables 1-2 and 1-3. The values in these tables should be rounded up to significant figures. Flow-frequency curves are shown in Figure 1-3.

TABLE 1-2

CALABAZAS CRK.-RECOMMENDED DESIGN FLOWS

1% FLOOD

RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA	
HYDROGRAPH AT	1	1179.	718.	408.	316.	2.85	
ROUTED TO	1	1179.	718.	408.	316.	2.85	U/S PROSPECT CRK
HYDROGRAPH AT	1	634.	350.	193.	149.	1.44	
ROUTED TO	1	634.	350.	193.	149.	1.44	PROSPECT CRK U/S CALABAZAS CRK
2 COMBINED	1	1811.	1067.	600.	465.	4.29	D/S PROSPECT CRK
ROUTED TO	1	1569.	1060.	599.	465.	4.29	
HYDROGRAPH AT	2	123.	80.	42.	33.	0.38	
HYDROGRAPH AT	2	198.	145.	93.	74.	0.58	
2 COMBINED	2	301.	225.	135.	107.	0.96	
ROUTED TO	2	219.	208.	135.	107.	0.96	
2 COMBINED	2	1875.	1251.	734.	571.	5.25	U/S RODEO CRK
HYDROGRAPH AT	2	285.	201.	110.	85.	0.86	
HYDROGRAPH AT	2	208.	152.	97.	76.	0.57	
2 COMBINED	2	460.	353.	208.	163.	1.43	
ROUTED TO	2	317.	301.	207.	163.	1.43	
ROUTED TO	2	316.	301.	207.	163.	1.43	RODEO CRK U/S CALABAZAS CRK
2 COMBINED	2	2149.	1525.	939.	734.	6.68	D/S RODEO CRK
ROUTED TO	2	2067.	1523.	938.	734.	6.69	
HYDROGRAPH AT	3	506.	339.	183.	142.	1.57	
HYDROGRAPH AT	3	623.	461.	295.	235.	1.84	
2 COMBINED	3	1055.	799.	478.	376.	3.41	
ROUTED TO	3	694.	643.	473.	376.	3.41	
ROUTED TO	3	693.	643.	473.	376.	3.41	REGNART CRK U/S CALABAZAS CRK
2 COMBINED	3	2646.	2127.	1407.	1110.	10.09	D/S REGNART CRK
ROUTED TO	3	2611.	2125.	1406.	1110.	10.09	
HYDROGRAPH AT	4	119.	73.	40.	31.	0.37	
HYDROGRAPH AT	4	352.	258.	165.	132.	1.13	
2 COMBINED	4	448.	331.	205.	162.	1.50	
ROUTED TO	4	424.	330.	205.	162.	1.50	
2 COMBINED	4	3036.	2450.	1609.	1272.	11.59	AT I-280
ROUTED TO	4	2879.	2433.	1603.	1272.	11.59	
HYDROGRAPH AT	5	191.	120.	63.	49.	0.65	
HYDROGRAPH AT	5	374.	299.	193.	157.	1.46	
2 COMBINED	5	544.	417.	256.	206.	2.11	
ROUTED TO	5	257.	256.	238.	203.	2.11	
2 COMBINED	5	3133.	2686.	1831.	1474.	13.70	AT EL CAMINO REAL
ROUTED TO	5	3114.	2684.	1331.	1472.	13.70	
HYDROGRAPH AT	6	356.	232.	124.	96.	1.29	
HYDROGRAPH AT	6	548.	398.	252.	200.	1.94	
2 COMBINED	6	845.	629.	376.	296.	3.23	
ROUTED TO	6	663.	620.	375.	296.	3.23	
ROUTED TO	6	663.	619.	375.	296.	3.23	EL CAMINO STORM DRAIN U/S CALABAZAS CRK
2 COMBINED	6	3777.	3292.	2200.	1769.	16.93	AT SPRR
ROUTED TO	6	3606.	3245.	2183.	1757.	16.93	
HYDROGRAPH AT	7	305.	193.	102.	79.	1.16	
HYDROGRAPH AT	7	684.	497.	314.	250.	2.58	
2 COMBINED	7	931.	689.	416.	329.	3.74	
ROUTED TO	7	684.	665.	415.	329.	3.74	
2 COMBINED	7	4288.	3577.	2589.	2036.	20.67	AT SLOUGH

TABLE 1-3

CALABAZAS CRK.-RECOMMENDED DESIGN FLOWS

10% FLOOD

RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA	
HYDROGRAPH AT	1	583.	336.	204.	159.	2.85	
ROUTED TO	1	583.	336.	204.	159.	2.85	U/S PROSPECT CRK
HYDROGRAPH AT	1	304.	154.	93.	72.	1.44	
ROUTED TO	1	304.	154.	93.	72.	1.44	PROSPECT CRK U/S CALABAZAS CRK
2 COMBINED	1	875.	491.	296.	231.	4.29	D/S PROSPECT CRK
ROUTED TO	1	773.	488.	295.	231.	4.29	
HYDROGRAPH AT	2	78.	47.	21.	16.	0.38	
HYDROGRAPH AT	2	134.	96.	60.	48.	0.58	
2 COMBINED	2	197.	143.	81.	64.	0.96	
ROUTED TO	2	186.	143.	81.	64.	0.96	
2 COMBINED	2	958.	528.	376.	294.	5.25	U/S RODEO CRK
HYDROGRAPH AT	2	176.	118.	55.	42.	0.86	
HYDROGRAPH AT	2	141.	101.	63.	50.	0.57	
2 COMBINED	2	293.	219.	117.	92.	1.43	
ROUTED TO	2	256.	217.	117.	92.	1.43	
ROUTED TO	2	256.	216.	117.	92.	1.43	RODEO CRK U/S CALABAZAS CRK
2 COMBINED	2	1194.	837.	491.	386.	6.68	D/S RODEO CRK
ROUTED TO	2	1141.	835.	491.	386.	6.68	
HYDROGRAPH AT	3	313.	196.	88.	67.	1.57	
HYDROGRAPH AT	3	425.	305.	190.	151.	1.84	
2 COMBINED	3	683.	500.	277.	218.	3.41	
ROUTED TO	3	519.	477.	276.	218.	3.41	
ROUTED TO	3	518.	476.	276.	218.	3.41	REGNART CRK U/S CALABAZAS CRK
2 COMBINED	3	1624.	1298.	766.	604.	10.09	D/S REGNART CRK
ROUTED TO	3	1605.	1296.	766.	604.	10.09	
HYDROGRAPH AT	4	74.	42.	19.	15.	0.37	
HYDROGRAPH AT	4	238.	171.	106.	84.	1.13	
2 COMBINED	4	295.	213.	125.	99.	1.50	
ROUTED TO	4	291.	212.	125.	99.	1.50	
2 COMBINED	4	1878.	1504.	891.	703.	11.59	AT I-280
ROUTED TO	4	1779.	1489.	888.	703.	11.59	
HYDROGRAPH AT	5	119.	71.	33.	25.	0.65	
HYDROGRAPH AT	5	247.	196.	125.	101.	1.46	
2 COMBINED	5	352.	265.	157.	126.	2.11	
ROUTED TO	5	243.	240.	157.	126.	2.11	
2 COMBINED	5	2021.	1729.	1045.	829.	13.70	AT EL CAMINO REAL
ROUTED TO	5	2012.	1727.	1045.	829.	13.70	
HYDROGRAPH AT	6	215.	131.	59.	45.	1.29	
HYDROGRAPH AT	6	363.	261.	162.	129.	1.94	
2 COMBINED	6	538.	391.	221.	174.	3.23	
ROUTED TO	6	526.	390.	221.	174.	3.23	
ROUTED TO	6	523.	389.	221.	174.	3.23	EL CAMINO STORM DRAIN U/S CALABAZAS CRK
2 COMBINED	6	2503.	2103.	1265.	1003.	16.93	AT SPRR
ROUTED TO	6	2362.	2068.	1258.	1000.	16.93	
HYDROGRAPH AT	7	191.	115.	53.	41.	1.16	
HYDROGRAPH AT	7	454.	326.	203.	161.	2.58	
2 COMBINED	7	604.	440.	256.	202.	3.74	
ROUTED TO	7	585.	438.	256.	202.	3.74	
2 COMBINED	7	2792.	2465.	1509.	1202.	20.67	AT SLOUGH

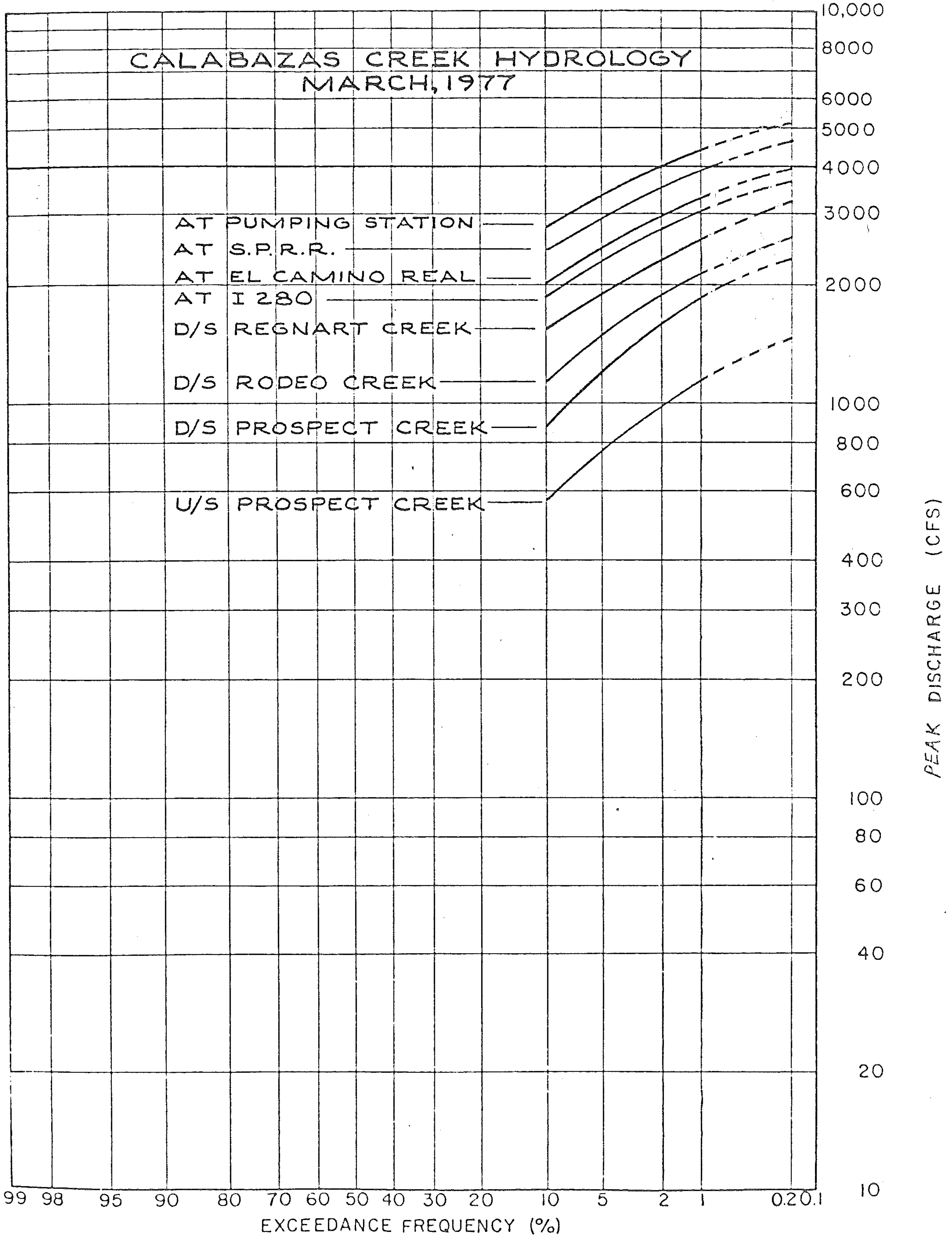

CALABAZAS CREEK HYDROLOGY
MARCH, 1977
AT PUMPING STATION
AT S.P.R.R.
AT EL CAMINO REAL
AT I 280
D/S REGNART CREEK
D/S RODEO CREEK
D/S PROSPECT CREEK
U/S PROSPECT CREEK
10,000
8000
6000
5000
4000
3000
2000
1000
800
600
400
300
200
100
80
60
40
20
10
PEAK DISCHARGE (CFS)
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.2
0.1
EXCEEDANCE FREQUENCY (%)

FIGURE 1-4

SARATOGA - SAN TOMAS CREEKS

HYDROLOGY

DETERMINATION OF DESIGN FLOWS

June 1978

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GENERAL INFORMATION

San Tomas Creek originates in the lower ranges of the Santa Cruz Mountains. The creek drains a total of 40 square miles before emptying into San Francisco Bay. Saratoga Creek (Figure 2-1) is the major tributary of San Tomas Creek (Figure 2-2) and has 16.5 square miles of drainage area at its confluence with the main stream just above Monroe Street in Santa Clara. Over half of the total watershed area is on the valley floor and has undergone extensive urbanization. The basin receives an average annual rainfall ranging from 12 inches to over 50 inches in the uppermost reaches of the basin where the elevation is over 3,000 feet. There are no reservoirs on either San Tomas or Saratoga Creeks which function as a flood control facility.

The objective of this report is to document the hydrology work done to determine design flows. It will discuss the use of regional regression equations and the analysis of recorded data, together with the special treatment of urban areas.

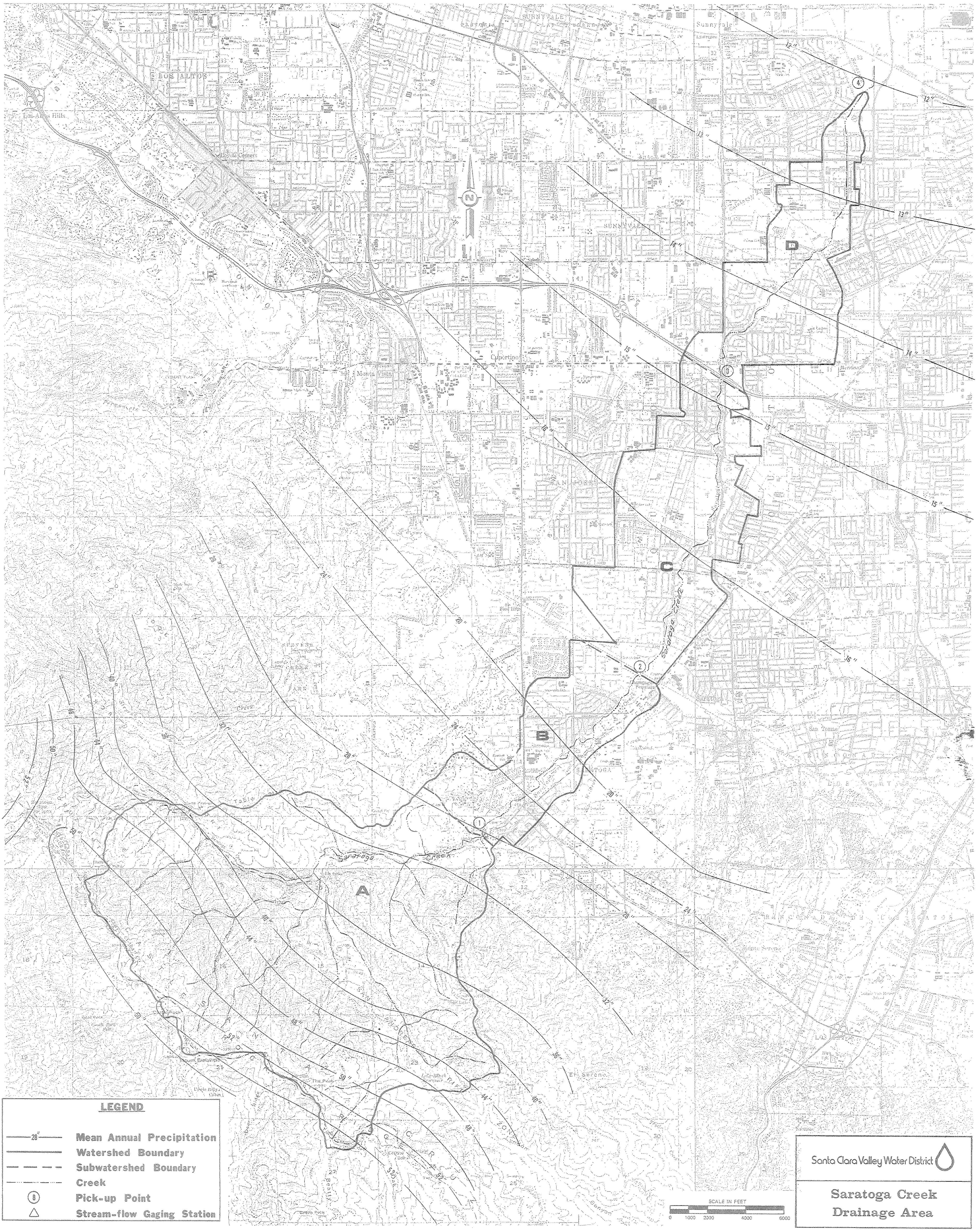
LEGEND
28" Mean Annual Precipitation
Watershed Boundary
Subwatershed Boundary
Creek
⓪ Pick-up Point
△ Stream-flow Gaging Station
Santa Clara Valley Water District
Saratoga Creek Drainage Area
SCALE IN FEET
0 1000 2000 4000 6000
N
A
B
C
D
①
②
③
④
12"
13"
14"
15"
16"
20"
24"
28"
32"
36"
40"
44"
48"
50"
52"
Saratoga Creek
Mountain View
LOS ALTOS
Los Altos Hills
SUNNYVALE
Cupertino
Monta Vista
SAN JOSE
SARATOGA
Monte Sereno
El Sereno

Figure 2-1

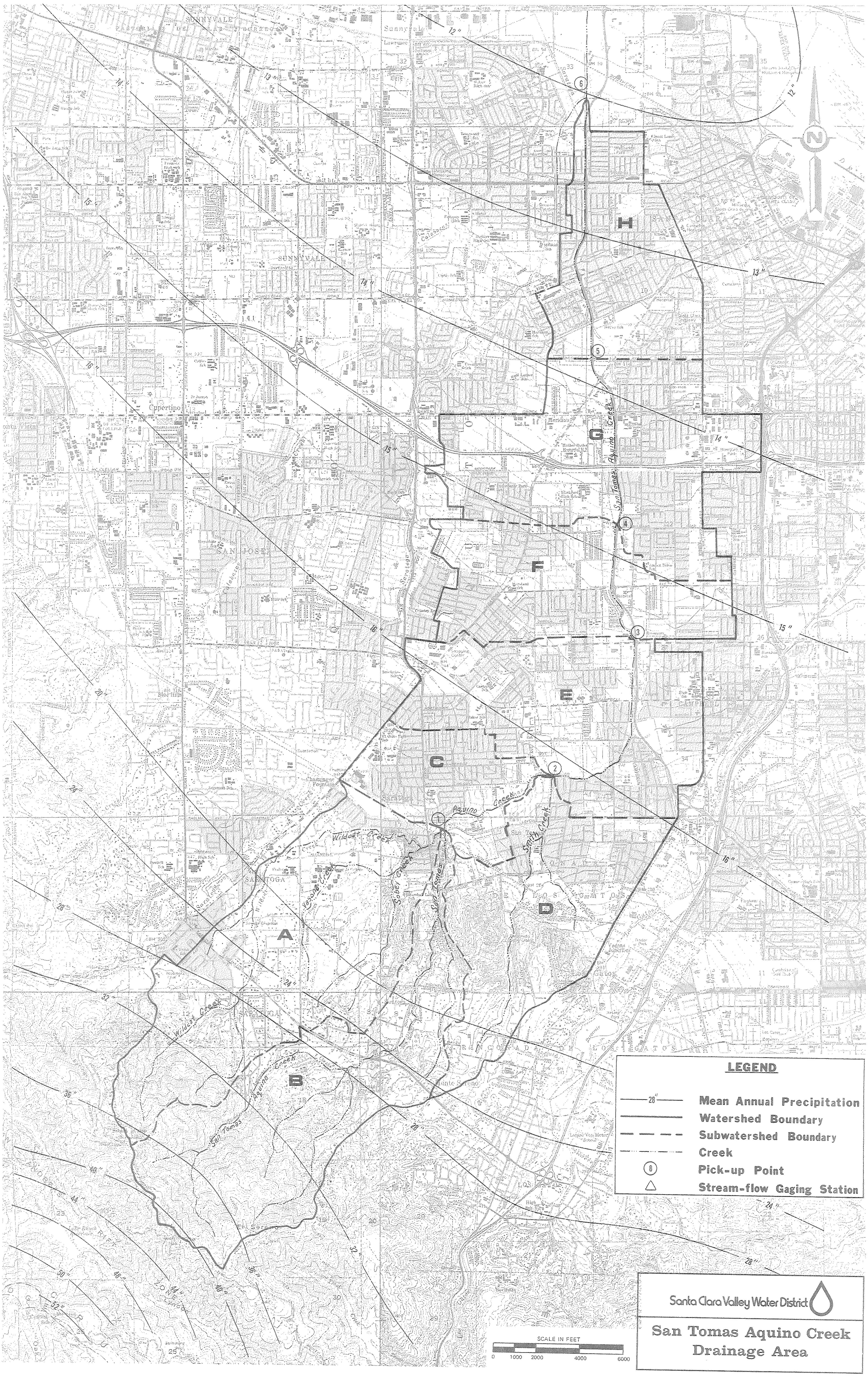LEGEND
28" Mean Annual Precipitation
Watershed Boundary
Subwatershed Boundary
Creek
⑥ Pick-up Point
△ Stream-flow Gaging Station
Santa Clara Valley Water District
San Tomas Aquino Creek Drainage Area
SCALE IN FEET
0
1000
2000
4000
6000
N
A
B
C
D
E
F
G
H
①
②
③
④
⑤
⑥
San Tomas Aquino Creek
Aquino Creek
Smith Creek
Wildcat Creek
Vasona Creek
Sobey Creek
San Tomas
12"
13"
14"
15"
16"
20"
24"
28"
32"
36"
40"
44"
50"
52"
SUNNYVALE
SAN JOSE
SARATOGA
LOS GATOS
Cupertino
Monte Sereno

Figure 2-2

DISCUSSION OF PROCEDURES

Basin Characteristics and 1% Regional Flows

The Saratoga-San Tomas Creeks Watershed is divided into four major watersheds: Saratoga rural, which includes subarea A_1; Saratoga urban, which includes subareas B_1, C_1, and D_1; San Tomas rural, which includes subareas A_2, B_2, and D_2; and San Tomas urban, which includes subareas C_2, E, F, G, and H (Figures 2-1 and 2-2). The subareas have the following basin characteristics:

Location	Sub-Area	*Drainage Area (Sq. Mi.)	Mean Annual Precip. (in.)	Lls $(mi^2/ft/ft^{1/2})$	Tc (Hours)
Saratoga Crk @ USGS Gage	A_1	9.20	42	64.9	1.0
Saratoga Crk @ SPRR	B_1	AI - .98 AP - .98	22	-	.5 2.5
Saratoga Crk @ Freeway 280	C_1	AI - 1.90 AP - 1.26	16	-	.5 2.5
Saratoga Crk U/S San Tomas Creek	D_1	AI - 1.34 AP - .90	13.5	-	.5 2.5
Wildcat Crk U/S San Tomas Creek	A_2	4.12	24	68.2	1.1
San Tomas Creek U/S Wildcat Crk	B_2	3.64	29	75.4	1.2
San Tomas Crk U/S Smith Creek	C_2	AI - .74 AP - .49	17	-	.5 2.5
Smith Creek U/S San Tomas Creek	D_2	2.65	18	33.2	1.2
San Tomas Creek @ Hamilton Ave.	E	AI - 1.85 AP - 1.23	16	-	.4 2.4
San Tomas Creek @ Williams Road	F	AI - 1.31 AP - .88	15	-	.4 2.4
San Tomas Creek @ Pruneridge Ave.	G	AI - 2.09 AP - 1.40	14.5	-	.4 2.4
San Tomas Crk U/S Saratoga Creek	H	AI - 1.26 AP - .84	13	-	.4 2.4

*A1 = Impervious Urban Area
AP = Pervious Urban Area
LLS = Basin Factor

In rural areas, the preliminary evaluation of design flows from regional regression equations is determined using the drainage areas, mean annual precipitation and basin factors. These preliminary values are compared with peak flows obtained from rainfall by employing the unit hydrograph theory and HEC-1 program. Although this latter technique is used mainly to define design hydrographs, the resultant peak flow is compared and balanced against the regional one. As will be shown later, the end result of the above procedure is further compared with results from the site specific analysis of recorded data if available.

Urban areas are treated based on the District's method of urban hydrology. The density of land use is determined by the ratio of impervious to pervious parts.

Recorded Data and Application of Regional Regression Equations

Inherent in the District hydrology method are checking procedures that utilize additional data wherever available. Stream flow records, historic data, stochastically-generated data, or data obtained from rainfall are utilized. Following is a discussion of all of these procedures and their application to Saratoga and San Tomas Creeks.

Stream Flow Records

A statistical analysis of maximum annual peak discharge data is the primary basis for the determination of the flow-frequency curve.

There are a total of six stream flow gaging stations in the watershed of Saratoga and San Tomas Creeks. One gage is operated by the U.S.G.S. and the other five are operated by the Santa Clara Valley Water District.

SCVWD #24	San Tomas Creek @ Williams Road
SCVWD #25	Saratoga Creek @ Pruneridge Avenue
SCVWD #27	Smith Creek below Elam Avenue
SCVWD #29	San Tomas Creek @ SPRR
SCVWD #30	Wildcat Creek @ SPRR
U.S.G.S.	Saratoga Creek @ Saratoga

All of these records were examined during the course of this hydrologic investigation and only the U.S.G.S. station was used. The five stations owned by the SCVWD have, until recent years, been operated primarily as water supply stations, concentrating on low flow volumes rather than the instantaneous peaks necessary for flood studies.

Saratoga and San Tomas Creeks have both experienced high flows in past years. Since 1934 flood flows have been recorded by the U. S. Geological Survey stream gage on Saratoga Creek in Saratoga. The maximum annual peaks from 1934 to present are:

1.	2730 cfs	December 22, 1955
2.	2540 cfs	February 27, 1940
3.	1650 cfs	January 21, 1943
4.	1580 cfs	January 16, 1973
5.	1240 cfs	January 12, 1952
6.	501 cfs	January 14, 1978

The U.S.G.S. stream gage at Saratoga (area = 9.2 square miles) has been recording flows continuously since 1934. This gage has the longest, continuous, homogeneous (unaffected by reservoirs or urbanization) data in the County. This station has also been used as one of the base stations in all regional frequency regression studies.

Analysis of the 42 years of stream flow records produced the following statistics:

$M = 2.621$

$O = .379$

$g = +.254$

1%= 3370 cfs

The skew coefficient is difficult to determine reliably for stream gages which have a short period of record. The Water Resources Council (Ref. 2) recommends weighting the skew using the following formula:

$$g = \frac{(N-25)}{75}\ \text{Recorded} + \left(1 - \frac{N-25}{75}\right)\ \text{Regional}$$

If $N \leq 25$, the regional skew (-.6) would be used and if $N \leq 100$, the recorded skew would be used.

Following the recommendation of the Water Resources Council, the weighted skew for the 42 years of record would be $g = -.41$ which gives a 1% flow rate of 2,443 cfs. Each annual peak and the computed frequency curve is plotted in Figure 2-1). Since this flow rate has been exceeded twice (1940 and 1955) in

TABLE 2-1

Maximum Annual Peak Flows 1915*- 1975
U.S.G.S. Gage Saratoga Creek @ Saratoga

Year	Peak	Year	Peak
1915	253. Est.*	1946	287.
1916	1619. Est.	1947	100.
1917	434. Est.	1948	134.
1918	345. Est.	1949	293.
1919	189. Est.	1950	222.
1920	387. Est.	1951	826.
1921	459. Est.	1952	1240.
1922	806. Est.	1953	494.
1923	48. Est.	1954	232.
1924	543. Est.	1955	107.
1925	160. Est.	1956	2730.
1926	346. Est.	1957	225.
1927	858. Est.	1958	772.
1928	616. Est.	1959	683.
1929	451. Est.	1960	178.
1930	670. Est.	1961	129.
1931	187. Est.	1962	432.
1932	334. Est.	1963	1160.
1933	103. Est.	1964	338.
1934	314.	1965	535.
1935	254.	1966	151.
1936	268.	1967	583.
1937	910.	1968	598.
1938	611.	1969	1120.
1939	110.	1970	406.
1940	2540.	1971	255.
1941	608.	1972	127.
1942	620.	1973	1580.
1943	1650.	1974	345.
1944	175.	1975	398.
1945	898.		

*Peak flow rates for 1915-1933 were established using multiple cross correlation with surrounding stations as part of 1976 Regional Regression Study. The rest of the flows are obtained from U.S.G.S. Water Supply papers.

the 42 years of recorded data, it is not considered realistic. The following sections include further investigations of this problem were as estimated stochastic records were used to enhance the information available in the recorded data.

Comparisons with Similar Watersheds

Comparisons between frequency curves and maximum flood data of the watershed being investigated and those in a hydrologically similar region are useful for identification of unusual events and testing the reasonableness of flood flow-frequency determinations.

The length and reliability of the records at the Saratoga Creek gage have enabled it to be used as a base station in the regional regression analysis. The 42 years of record were correlated in a matrix with data from 20 other stream flow stations with up to 60 years of record. The correlation enhanced the Saratoga gage statistics to be equivalent to 52 years of record. Recorded and estimated annual maximums are compiled in Table 2-1. Using the mean and standard deviation calculated from the 20-station/60-year data matrix and the weighted skewness of -.41 the 1% flood peak was estimated at 2,163 cfs. This last value is still considered low with respect to recorded data.

Flood Estimates from Precipitation

Flood flows in streams are affected by the combinations of prior rainfall, soil condition, and available storage capacity. Under proper conditions, a storm of a certain intensity may cause a flood of larger size than what is expected. This is obvious as the dates of the maximum recorded one-day* rainfalls, compiled in Table 2-2 for several precipitation stations in the San Tomas drainage, do not necessarily fall on the same days as the larger floods.

---

*One-day rainfall was used as it was readily available. The maximum 24-hour rainfall would be approximately 13% higher.

The mean annual precipitation (M.A.P.) reflects well the integrated effects of availability of atmospheric moisture, of elevation and topography, and of efficiency of the orographic mechanism, which are significant parameters of storm rainfall. It was, therefore, possible to develop a relationship of storm volumes vs. M.A.P. for different durations and frequency using data collected from rainfall stations in and around the County. The 24-hour rain with an exceedance frequency of 1% is used as the design storm.

The unit hydrograph theory was employed to convert rainfall to runoff. Clark's synehetic unit hydrograph was used with the Corps of Engineers' 24-hour standard storm distribution and a 30% loss rate. It has been found that a 30% loss rate, applied to the one percent storm, closely approximates the one percent flood on watersheds with average hydrologic characteristics. The results of this procedure are compiled in the flow summary, Tables 2-3 and 2-4. Peak flow rate at the U.S.G.S. gaging station is 3,640 cfs using this approach. These are rough figures and a parameter optimization technique would be necessary to refine them further; however, they do provide a guide to prevent gross errors.

Table 2-2

Records of Rainfall
Saratoga - San Tomas Creeks

		Max. Annual Recorded 1-Day Rainfall		
#5	Alviso (Leslie Salt)	1	3.54	January 12, 1952
	Elev - 8	2	2.40	December 24, 1955
	1941-1974	3	2.20	November 29, 1970
#90	Santa Clara University	1	3.79	January 30, 1968
	Elev - 88	2	3.42	October 13, 1962
	1951-1974	3	3.29	December 23, 1955
		4	127.	1978
#14	Campbell Water Company	1	4.82	January 30, 1968
	Elev - 192	2	4.55	December 23, 1955
	1936-1974	3	3.70	October 13, 1962
		4	44.5	
#20	Cox Avenue	1	4.70	December 22, 1955
	Elev - 325	2	4.26	January 21, 1967
	1955-1974	3	4.21	January 29, 1968
		4	40.92	March 5, 1978
#94	Saratoga Gap	1	11.18	December 23, 1955
	Elev - 2600	2	8.94	November 19, 1950
	1937-1974	3	8.88	February 27, 1940
		4	127.	

TABLE 2-3

SARATOGA CREEK

RAINFALL GENERATED FLOWS

RUNOFF SUMMARY, AVERAGE FLOW

			PEAK	6-HOUR	24-HOUR	50-HOUR	AREA
USGS Stream Gage	HYDROGRAPH AT	1	3636.	2967.	1877.	938.	9.20
	ROUTED TO	2	3603.	2967.	1876.	938.	9.20
	HYDROGRAPH AT	2	487.	293.	184.	89.	0.98
	HYDROGRAPH AT	2	343.	242.	136.	66.	0.98
	2 COMBINED	2	769.	529.	319.	155.	1.96
	ROUTED TO	2	256.	255.	232.	155.	1.96
@ SPRR	2 COMBINED	2	3843.	3210.	2101.	1094.	11.16
	ROUTED TO	3	3865.	3206.	2101.	1094.	11.16
	HYDROGRAPH AT	3	788.	469.	291.	140.	1.90
	HYDROGRAPH AT	3	368.	256.	142.	69.	1.26
	2 COMBINED	3	1092.	716.	431.	209.	3.16
	ROUTED TO	3	380.	378.	348.	209.	3.16
@ 280 Freeway	2 COMBINED	3	4234.	3573.	2444.	1303.	14.32
	ROUTED TO	4	4195.	3573.	2444.	1303.	14.32
	HYDROGRAPH AT	4	511.	301.	184.	89.	1.34
	HYDROGRAPH AT	4	242.	167.	91.	44.	0.90
	2 COMBINED	4	709.	462.	274.	133.	2.24
	ROUTED TO	4	257.	256.	238.	133.	2.24
U/S San Tomas	2 COMBINED	4	4446.	3824.	2681.	1437.	16.56

TABLE 2-4
SAN TOMAS CREEK
RAINFALL GENERATED FLOWS

RUNOFF SUMMARY, AVERAGE FLOW

			PEAK	6-HOUR	24-HOUR	50-HOUR	AREA
Wildcat U/S San Tomas	HYDROGRAPH AT	1	1523.	1043.	599.	292.	4.12
	ROUTED TO	1	1523.	1043.	599.	292.	4.12
U/S Wildcat Creek	HYDROGRAPH AT	1	1528.	1034.	600.	291.	3.64
	ROUTED TO	1	1528.	1034.	600.	291.	3.64
D/S Wildcat Creek	2 COMBINED	1	3052.	2076.	1199.	583.	7.76
	ROUTED TO	2	2963.	2076.	1199.	583.	7.76
	HYDROGRAPH AT	2	320.	190.	118.	57.	0.74
	HYDROGRAPH AT	2	149.	104.	58.	28.	0.49
	2 COMBINED	2	443.	290.	175.	85.	1.23
	ROUTED TO	2	151.	150.	138.	85.	1.23
U/S Smith Creek	2 COMBINED	2	3107.	2217.	1329.	668.	8.99
Smith Cr U/S San Tomas	HYDROGRAPH AT	2	834.	572.	327.	159.	2.65
	ROUTED TO	2	834.	572.	327.	159.	2.65
D/S Smith Creek	2 COMBINED	2	3940.	2789.	1655.	827.	11.64
	ROUTED TO	3	3917.	2787.	1656.	827.	11.64
	HYDROGRAPH AT	3	768.	456.	283.	137.	1.85
	HYDROGRAPH AT	3	363.	251.	139.	67.	1.23
	2 COMBINED	3	1067.	699.	420.	204.	3.08
	ROUTED TO	3	370.	368.	339.	204.	3.08
@ Hamilton Ave.	2 COMBINED	3	4277.	3138.	1981.	1032.	14.72
	ROUTED TO	4	4268.	3139.	1982.	1032.	14.72
	HYDROGRAPH AT	4	522.	310.	192.	93.	1.31
	HYDROGRAPH AT	4	249.	172.	95.	46.	0.88
	2 COMBINED	4	727.	476.	286.	139.	2.19
	ROUTED TO	4	258.	257.	237.	139.	2.19
@ Williams Road	2 COMBINED	4	4519.	3385.	2212.	1171.	16.91
	ROUTED TO	5	4463.	3385.	2213.	1171.	16.91
	HYDROGRAPH AT	5	815.	484.	300.	145.	2.09
	HYDROGRAPH AT	5	384.	268.	149.	72.	1.40
	2 COMBINED	5	1131.	743.	447.	217.	3.49
	ROUTED TO	5	409.	407.	376.	217.	3.49
@ Pruneridge Ave.	2 COMBINED	5	4861.	3775.	2582.	1388.	20.40
	ROUTED TO	6	4826.	3773.	2582.	1388.	20.40
	HYDROGRAPH AT	6	458.	272.	169.	82.	1.26
	HYDROGRAPH AT	6	215.	150.	83.	40.	0.84
	2 COMBINED	6	635.	416.	251.	122.	2.10
	ROUTED TO	6	239.	238.	222.	122.	2.10
U/S Saratoga Creek	2 COMBINED	6	5062.	4005.	2801.	1510.	22.50
Saratoga Creek	HYDROGRAPH AT	6	4446.	3824.	2681.	1437.	16.56
D/S Saratoga Creek	2 COMBINED	6	9467.	7812.	5479.	2947.	39.06

OTHER STUDIES - PAST AND PRESENT

The hydrology of San Tomas and Saratoga Creeks has been studied as different points several times in the past. Values currently listed in the District Manual of Design Flows (Ref. 7) are a conglomeration of application of several generations of regional regression equations.

The past regional equations, as developed in 1972, were used until revised in 1976. How well each set of equations represents the San Tomas Basin is shown graphically in Figure 2-2 where each flow-frequency curve is plotted in comparison with the recorded data at the Saratoga stream gage. The U.S. Geological Survey has developed its own set of regression equations for the San Francisco Bay Region (Ref. 8). The equations were used to develop the U.S.G.S. flow-frequency curve in Figure 2-2.

CONCLUSIONS

This report is the summary of all the work completed in the study of the Saratoga and San Tomas Creek hydrology.

The flow rates have been computed from the 1976 Regional Regression Equations; checked against statistical analysis of systematic streamflow records; compared with hydrologically similar watersheds; generated from rainfall; and contrasted with past studies of our own and of others.

The 1% statistical flows (2,443 cfs and 2,163 cfs) obtained from recorded, and recorded and estimated data, respectively, were exceeded twice in the 42 years of record. Such low values are the result of the application of a negative regional skew on data which is uniquely positively skewed.

The dilemma of having 1% flow estimates that have been exceeded twice within 42 years, indicates that the higher values obtained from regional equations should be considered with more weight. The regional 1% value (3,630 cfs) more nearly matches the 1% result of 3,370 cfs obtained from the statistical analysis of the data with the positive skew. Additionally, rainfall-generated flows produced results close to the regional ones. Therefore, the regional values coupled with the use of the urban method are recommended for the final determination of the design flows. This would provide homogeneity on ungaged locations.

The 1% design flows for Saratoga and San Tomas are summarized in Tables 2-5 and 2-6 along with the 10% design flows in Tables 2-7 and 2-8. The values in these tables should be rounded up to significant figures. Flow-frequency curves are shown in Figures 2-3 and 2-4.

TABLE 2-5

SARATOGA CREEK

RECOMMENDED DESIGN FLOWS -- 1% FLOOD

RUNOFF SUMMARY, AVERAGE FLOW

			PEAK	6-HOUR	24-HOUR	50-HOUR	AREA
USGS Stream Gage	HYDROGRAPH AT	1	3488.	2539.	1593.	796.	9.20
	ROUTED TO	2	3320.	2539.	1592.	796.	9.20
	HYDROGRAPH AT	2	487.	293.	184.	89.	0.98
	HYDROGRAPH AT	2	343.	242.	136.	66.	0.98
	2 COMBINED	2	769.	529.	319.	155.	1.96
	ROUTED TO	2	256.	255.	232.	155.	1.96
@ SPRR	2 COMBINED	2	3560.	2782.	1818.	952.	11.16
	ROUTED TO	3	3548.	2779.	1817.	952.	11.16
	HYDROGRAPH AT	3	788.	469.	291.	140.	1.90
	HYDROGRAPH AT	3	368.	256.	142.	69.	1.26
	2 COMBINED	3	1092.	716.	431.	209.	3.16
	ROUTED TO	3	380.	378.	348.	209.	3.16
@ 280 Fwy	2 COMBINED	3	3917.	3147.	2161.	1161.	14.32
	ROUTED TO	4	3833.	3146.	2161.	1161.	14.32
	HYDROGRAPH AT	4	511.	301.	184.	89.	1.34
	HYDROGRAPH AT	4	242.	167.	91.	44.	0.90
	2 COMBINED	4	709.	462.	274.	133.	2.24
	ROUTED TO	4	257.	256.	238.	133.	2.24
U/S San Tomas	2 COMBINED	4	4084.	3397.	2397.	1295.	16.56

TABLE 2-6
SAN TOMAS CREEK
RECOMMENDED DESIGN FLOWS - 1% FLOOD

RUNOFF SUMMARY, AVERAGE FLOW

			PEAK	6-HOUR	24-HOUR	50-HOUR	AREA
Wildcat U/S San Tomas	HYDROGRAPH AT	1	1590.	997.	569.	277.	4.12
	ROUTED TO	1	1590.	997.	569.	277.	4.12
U/S Wildcat Cr	HYDROGRAPH AT	1	1550.	999.	577.	280.	3.64
	ROUTED TO	1	1550.	999.	577.	280.	3.64
D/S Wildcat Cr	2 COMBINED	1	3140.	1995.	1146.	557.	7.76
	ROUTED TO	2	2969.	1995.	1145.	557.	7.76
	HYDROGRAPH AT	2	320.	190.	118.	57.	0.74
	HYDROGRAPH AT	2	149.	104.	58.	28.	0.49
	2 COMBINED	2	443.	290.	175.	85.	1.23
	ROUTED TO	2	151.	150.	138.	85.	1.23
U/S Smith Cr	2 COMBINED	2	3112.	2136.	1276.	642.	8.99
Smith U/S San Tomas	HYDROGRAPH AT	2	1030.	573.	322.	157.	2.65
	ROUTED TO	2	1030.	573.	322.	157.	2.65
D/S Smith Cr	2 COMBINED	2	4142.	2710.	1597.	799.	11.64
	ROUTED TO	3	3995.	2707.	1597.	799.	11.64
	HYDROGRAPH AT	3	768.	456.	283.	137.	1.85
	HYDROGRAPH AT	3	363.	251.	139.	67.	1.23
	2 COMBINED	3	1067.	699.	420.	204.	3.08
	ROUTED TO	3	370.	368.	339.	204.	3.08
@ Hamilton Ave	2 COMBINED	3	4354.	3059.	1923.	1003.	14.72
	ROUTED TO	4	4360.	3059.	1923.	1003.	14.72
	HYDROGRAPH AT	4	522.	310.	192.	93.	1.31
	HYDROGRAPH AT	4	249.	172.	95.	46.	0.88
	2 COMBINED	4	727.	476.	286.	139.	2.19
	ROUTED TO	4	258.	257.	237.	139.	2.19
@ Williams Rd	2 COMBINED	4	4611.	3306.	2155.	1142.	16.91
	ROUTED TO	5	4535.	3305.	2155.	1142.	16.91
	HYDROGRAPH AT	5	815.	484.	300.	145.	2.09
	HYDROGRAPH AT	5	384.	268.	149.	72.	1.40
	2 COMBINED	5	1131.	743.	447.	217.	3.49
	ROUTED TO	5	409.	407.	376.	217.	3.49
@ Pruneridge Ave	2 COMBINED	5	4933.	3695.	2524.	1360.	20.40
	ROUTED TO	6	4852.	3695.	2524.	1360.	20.40
	HYDROGRAPH AT	6	458.	272.	169.	82.	1.26
	HYDROGRAPH AT	6	215.	150.	83.	40.	0.84
	2 COMBINED	6	635.	416.	251.	122.	2.10
	ROUTED TO	6	239.	238.	222.	122.	2.10
U/S Saratoga Cr	2 COMBINED	6	5088.	3926.	2743.	1482.	22.50
Saratoga Cr	HYDROGRAPH AT	6	4084.	3397.	2397.	1295.	16.56
D/S Saratoga Cr	2 COMBINED	6	9127.	7306.	5138.	2776.	39.06

TABLE 2-7
SARATOGA CREEK
TEN-YEAR FLOOD FLOWS

RUNOFF SUMMARY, AVERAGE FLOW

			PEAK	6-HOUR	24-HOUR	50-HOUR	AREA
USGS Stream Gage	HYDROGRAPH AT	1	1937.	1636.	893.	439.	9.20
	ROUTED TO	2	1964.	1636.	892.	439.	9.20
	HYDROGRAPH AT	2	342.	199.	119.	58.	0.98
	HYDROGRAPH AT	2	219.	143.	67.	32.	0.98
	2 COMBINED	2	516.	337.	185.	90.	1.96
	ROUTED TO	2	214.	212.	178.	90.	1.96
@ SPRR	2 COMBINED	2	2172.	1845.	1070.	529.	11.16
	ROUTED TO	3	2156.	1841.	1069.	529.	11.16
	HYDROGRAPH AT	3	539.	313.	188.	91.	1.90
	HYDROGRAPH AT	3	229.	149.	70.	34.	1.26
	2 COMBINED	3	721.	456.	257.	124.	3.16
	ROUTED TO	3	334.	331.	252.	125.	3.16
@ 280 Fwy	2 COMBINED	3	2487.	2170.	1316.	653.	14.32
	ROUTED TO	4	2487.	2170.	1315.	653.	14.32
	HYDROGRAPH AT	4	341.	198.	119.	57.	1.34
	HYDROGRAPH AT	4	146.	96.	45.	22.	0.90
	2 COMBINED	4	457.	289.	163.	79.	2.24
	ROUTED TO	4	233.	231.	162.	79.	2.24
U/S San Tomas	2 COMBINED	4	2719.	2400.	1471.	732.	16.56

TABLE 2-8

SAN TOMAS CREEK

TEN-YEAR FLOOD FLOWS

RUNOFF SUMMARY, AVERAGE FLOW

			PEAK	6-HOUR	24-HOUR	50-HOUR	AREA
Wildcat U/S San Tomas	HYDROGRAPH AT	1	757.	546.	268.	130.	4.12
	ROUTED TO	1	757.	546.	268.	130.	4.12
U/S Wildcat Cr.	HYDROGRAPH AT	1	792.	587.	290.	140.	3.64
	ROUTED TO	1	792.	587.	290.	140.	3.64
D/S Wildcat Cr.	2 COMBINED	1	1548.	1133.	558.	269.	7.76
	ROUTED TO	2	1578.	1133.	558.	269.	7.76
	HYDROGRAPH AT	2	218.	127.	76.	37.	0.74
	HYDROGRAPH AT	2	93.	61.	29.	14.	0.49
	2 COMBINED	2	292.	185.	104.	50.	1.23
	ROUTED TO	2	131.	130.	102.	50.	1.23
U/S Smith Cr.	2 COMBINED	2	1707.	1260.	660.	320.	8.99
Smith U/S San Tomas	HYDROGRAPH AT	2	444.	284.	140.	68.	2.65
	ROUTED TO	2	444.	284.	140.	68.	2.65
D/S Smith Cr.	2 COMBINED	2	2151.	1544.	800.	388.	11.64
	ROUTED TO	3	2120.	1542.	798.	388.	11.64
	HYDROGRAPH AT	3	525.	305.	183.	88.	1.85
	HYDROGRAPH AT	3	226.	146.	68.	33.	1.23
	2 COMBINED	3	705.	445.	250.	121.	3.08
	ROUTED TO	3	326.	322.	246.	121.	3.08
@ Hamilton Ave	2 COMBINED	3	2443.	1860.	1043.	509.	14.72
	ROUTED TO	4	2455.	1860.	1043.	509.	14.72
	HYDROGRAPH AT	4	357.	207.	124.	60.	1.31
	HYDROGRAPH AT	4	155.	100.	47.	23.	0.88
	2 COMBINED	4	480.	304.	171.	83.	2.19
	ROUTED TO	4	230.	228.	169.	83.	2.19
@ Williams Rd	2 COMBINED	4	2684.	2086.	1210.	592.	16.91
	ROUTED TO	5	2673.	2086.	1210.	592.	16.91
	HYDROGRAPH AT	5	557.	324.	194.	94.	2.09
	HYDROGRAPH AT	5	239.	156.	73.	35.	1.40
	2 COMBINED	5	747.	473.	266.	129.	3.49
	ROUTED TO	5	366.	362.	263.	129.	3.49
@ Pruneridge Ave	2 COMBINED	5	3036.	2444.	1472.	721.	20.40
	ROUTED TO	6	3005.	2443.	1472.	721.	20.40
	HYDROGRAPH AT	6	312.	181.	109.	53.	1.26
	HYDROGRAPH AT	6	133.	87.	41.	20.	0.84
	2 COMBINED	6	418.	265.	149.	72.	2.10
	ROUTED TO	6	218.	215.	148.	72.	2.10
U/S Saratoga Cr	2 COMBINED	6	3222.	2656.	1618.	793.	22.50
Saratoga Cr	HYDROGRAPH AT	6	2719.	2401.	1471.	732.	16.56
D/S Saratoga Cr	2 COMBINED	6	5941.	5044.	3082.	1526.	39.06

September 1976

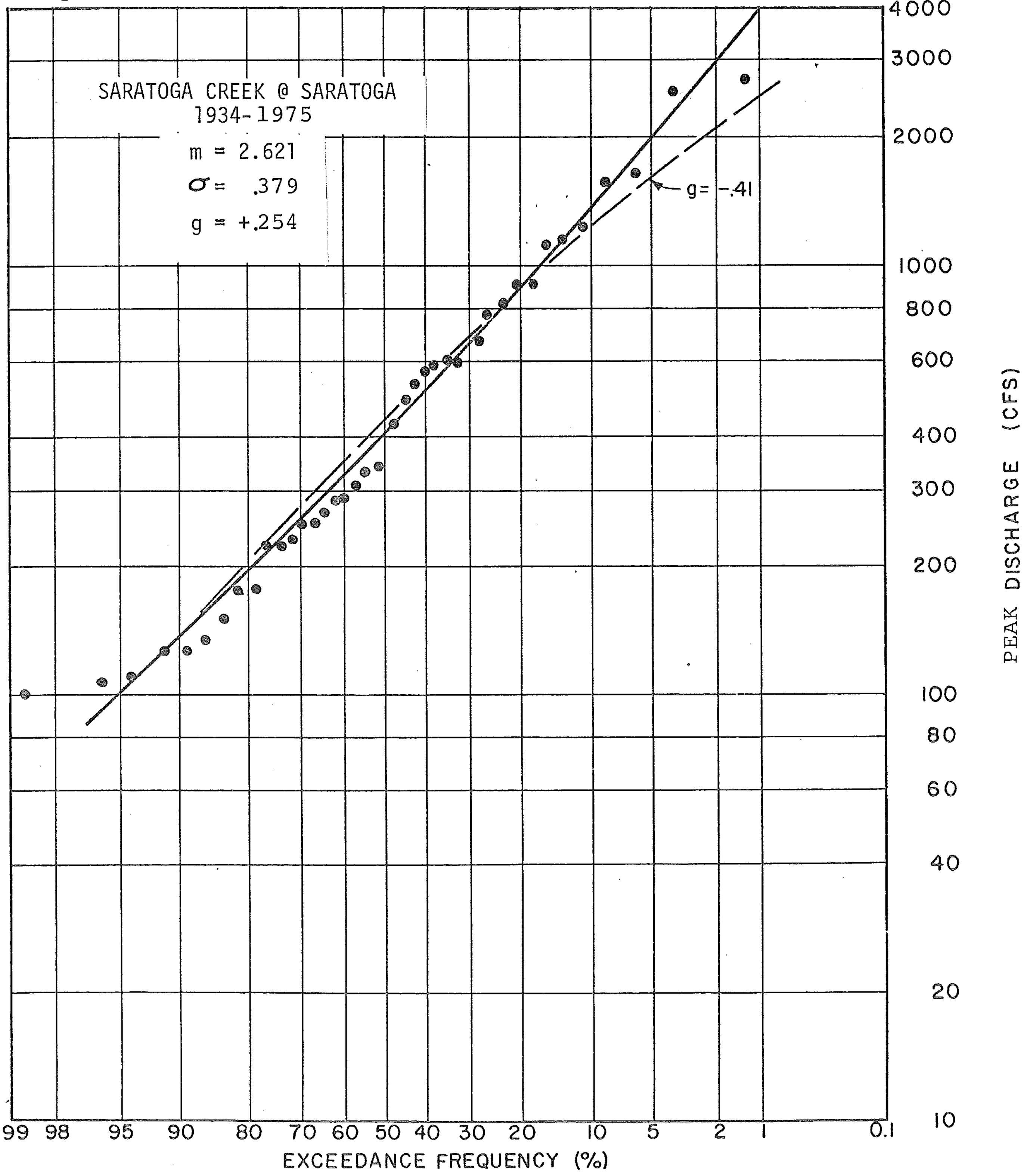

Figure 2-3

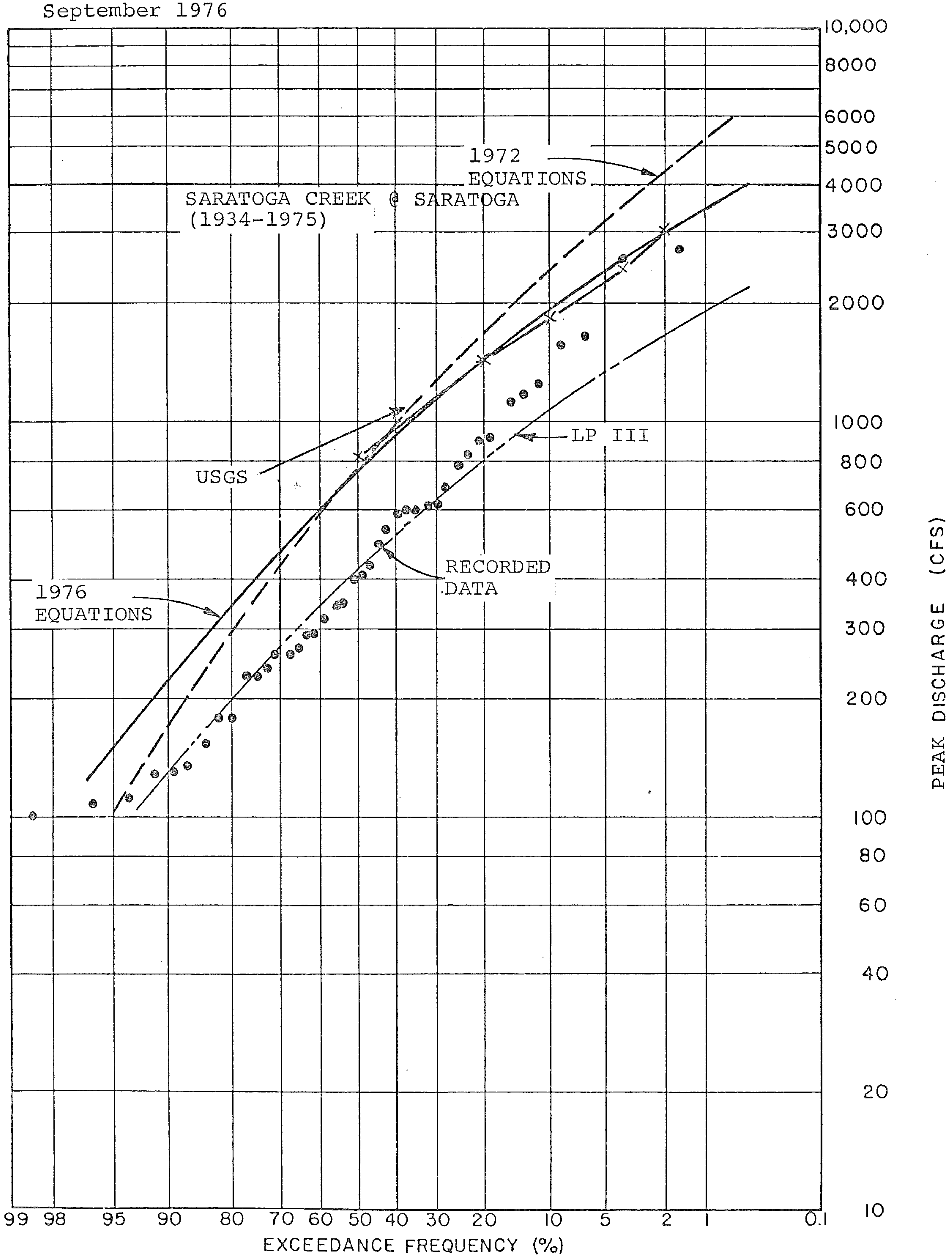
1972
EQUATIONS
SARATOGA CREEK @ SARATOGA
(1934-1975)
USGS
LP III
RECORDED
DATA
1976
EQUATIONS
10,000
8000
6000
5000
4000
3000
2000
1000
800
600
400
300
200
100
80
60
40
20
10
PEAK DISCHARGE (CFS)
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.1
EXCEEDANCE FREQUENCY (%)

Figure 2-4

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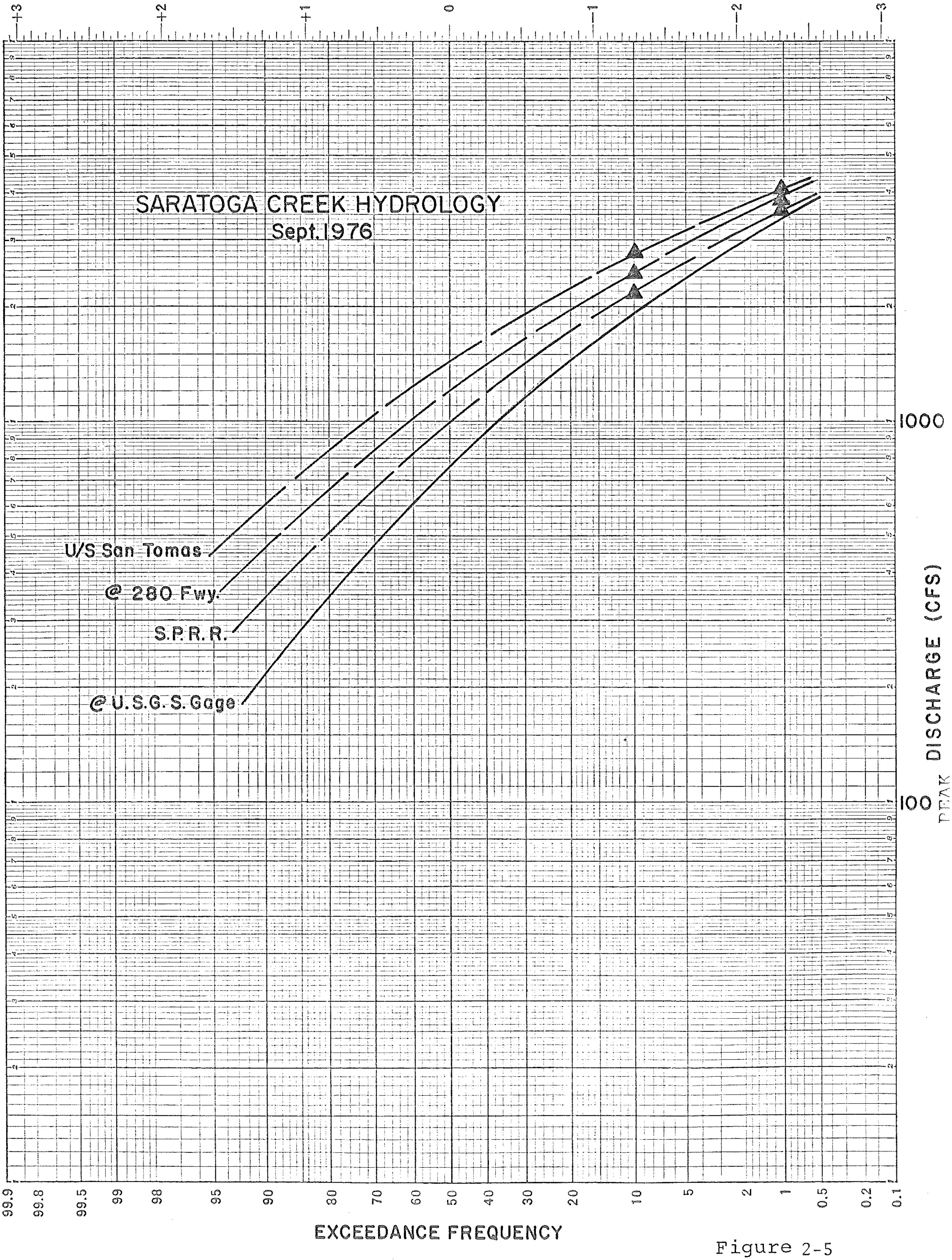

Figure 2-5

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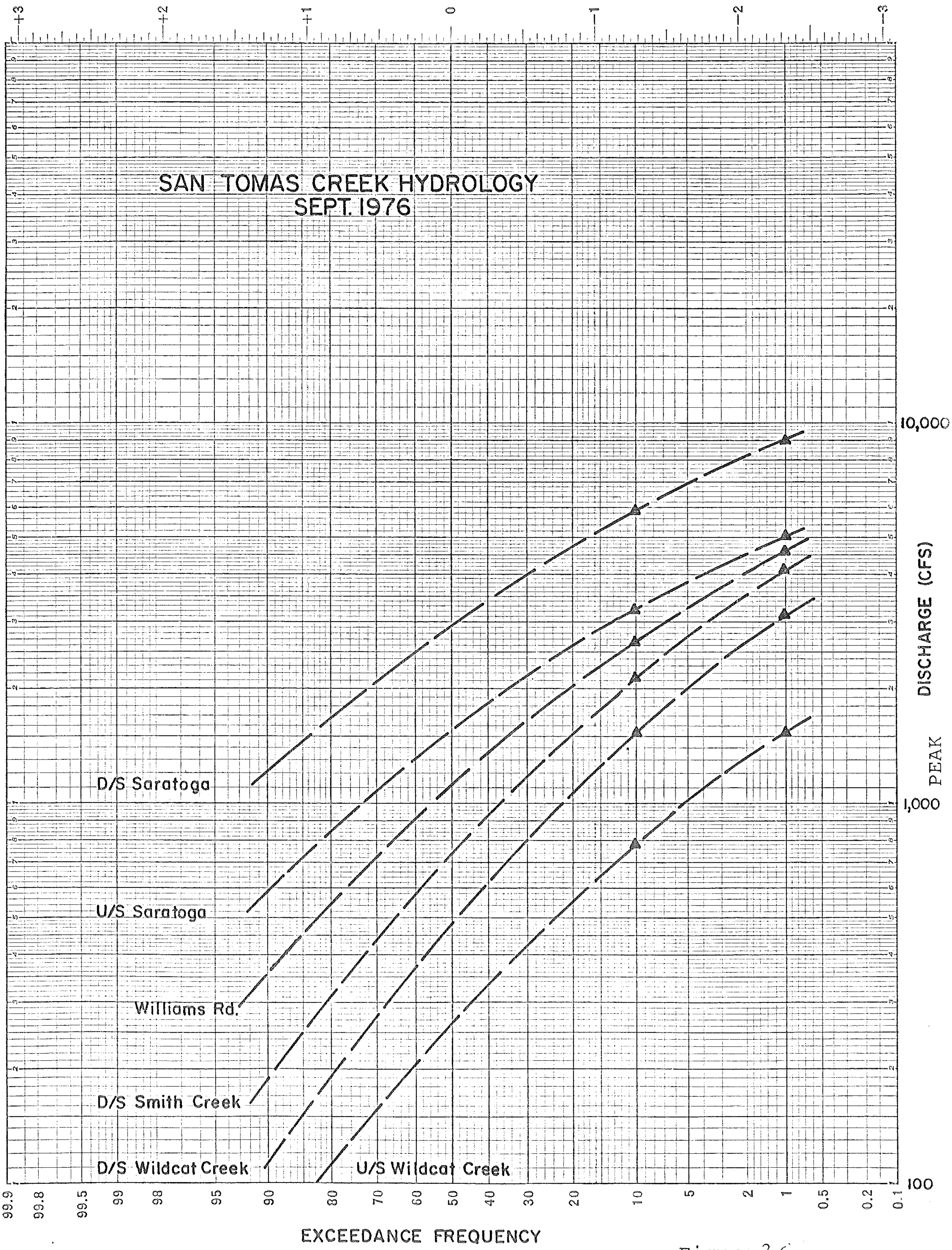

Figure 2-6

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